Chapter 4
Rock Mass Characterization

4-1. Scope

This chapter provides guidance in the description and engineering classification of intact rock and rock masses, the types, applications and analyses of rock property tests, the evaluation of intact rock and rock mass properties, and the selection of design parameters for project structures founded on rock. Rock mass characterization refers to the compilation of information and data to build a complete conceptual model of the rock foundation in which all geologic features that might control the stability of project structures, as well as the physical properties of those features, are identified and defined. The compilation of information and data is a continual process. The process starts with the preliminary site investigations and is expanded and refined during site exploration, laboratory and field testing, design analyses, construction and, in some cases, operation of the project structure. The order of information and data development generally reflects a district's approach to the process but usually evolves from generalized information to the specific details required by the design process. Furthermore, the level of detail required is dependent upon the project structure and the rock mass foundation conditions. For these reasons, this chapter is subdivided into five topic areas according to types of information rather than according to a sequence of tasks. Topic areas include geologic descriptions, engineering classification, shear strength parameters, bearing capacity parameters, and deformation and settlement parameters. The five topic areas provide required input to the analytical design processes described in Chapters 5, 6, 7, and 8.

4-2. Intact Rock versus Rock Mass

The in-situ rock, or rock mass, is comprised of intact blocks of rock separated by discontinuities such as joints, bedding planes, folds, sheared zones and faults. These rock blocks may vary from fresh and unaltered rock to badly decomposed and disintegrated rock. Under applied stress, the rock mass behavior is generally governed by the interaction of the intact rock blocks with the discontinuities. For purposes of design analyses, behavioral mechanisms may be assumed as discontinuous (e.g. sliding stability) or continuous (e.g. deformation and settlement).

Section I
Geologic Descriptions

4-3. General

Geologic descriptions contain some especially important qualitative and quantitative descriptive elements for intact rock and rock masses. Such descriptors are used primarily for geologic classification, correlation of stratigraphic units, and foundation characterization. A detailed description of the foundation rock, its structure, and the condition of its discontinuities can provide valuable insights into potential rock mass behavior. Geologic descriptors can, for convenience of discussion, be divided into two groups: descriptors commonly used to describe rock core obtained during site exploration core boring and supplemental descriptors required for a complete description of the rock mass. Descriptive elements are often tailored to specific geologic conditions of interest. In addition to general geologic descriptors, a number of rock index tests are frequently used to aid in geologic classification and characterization.

4-4. Rock Core Descriptors

Rock core descriptors refer to the description of apparent characteristics resulting from a visual and physical inspection of rock core. Rock core descriptors are recorded on the drilling log (ENG Form 1836) either graphically or by written description. Descriptions are required for the intact blocks of rock, the rock mass structure (i.e., fractures and bedding) as well as the condition and type of discontinuity. Criteria for the majorities of these descriptive elements are contained in Table B-2 of EM 1110-1-1804, Table 3-5 of EM 1110-1-1806, and Murphy (1985). Table 4-1 summarizes, consolidates, and, in some instances, expands descriptor criterion contained in the above references. Figures D-6 and D-7 of EM 1110-1-1804 provide examples of typical rock core logs. The following discussions provide a brief summary of the engineering significance associated with the more important descriptors.

a. Unit designation. Unit designation is usually an informal name assigned to a rock unit that does not necessarily have a relationship to stratigraphic rank (e.g. Miami oolite or Chattanooga shale).

Table 4-1
Summary of Rock Descriptors

1. Intact Blocks of Rock

a. Degree of Weathering.

(1) Unweathered: No evidence of any chemical or mechanical alteration.

(2) Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10 percent of the rock volume altered.

(3) Moderately weathered: Discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering “halos” evident, 10 to 50 percent of the rock altered.

(4) Highly weathered: Entire mass discolored, alteracation pervading nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away.

(5) Decomposed: Rock reduced to a soil with relicit rock texture, generally molded and crumbled by hand.

b. Hardness.

(1) Very soft: Can be deformed by hand.

(2) Soft: Can be scratched with a fingernail.

(3) Moderately hard: Can be scratched easily with a knife.

(4) Hard: Can be scratched with difficulty with a knife.

(5) Very hard: Cannot be scratched with a knife.

c. Texture.

(1) Sedimentary rocks:

Texture	Grain Diameter	Particle Name	Rock Name
*	80 mm	cobble	conglomerate
*	5 - 80 mm	gravel	
Coarse grained	2 - 5 mm		
Medium grained	0.4 - 2 mm	sand	sandstone
Fine grained	0.1 - 0.4 mm		
Very fine grained	0.1 mm	clay, silt	shale, claystone, siltstone

* Use clay-sand texture to describe conglomerate matrix.

(2) Igneous and metamorphic rocks:

Texture	Grain Diameter
Coarse grained	5 mm
Medium grained	1 - 5 mm
Fine grained	0.1 - 1 mm
Aphanite	0.1 mm

(Continued)

Table 4-1. (Continued)

(3) Textural adjectives: Use simple standard textural adjectives such as prophyritic, vesicular, pegmatitic, granular, and grains well developed, but not sophisticated terms such as holohyaline, hypidimorphic granular, crystal loblastic, and cataclastic.

d. Lithology Macro Description of Mineral Components.

Use standard adjectives such as shaly, sandy, silty, and calcareous. Note inclusions, concretions, nodules, etc.

2. Rock Structure

a. Thickness of Bedding.

(1) Massive: 3-ft thick or greater.

(2) Thick bedded: beds from 1- to 3-ft thick.

(3) Medium bedded: beds from 4 in. to 1-ft thick.

(4) Thin bedded: 4-in. thick or less.

b. Degree of Fracturing (Jointing).

(1) Unfractured: fracture spacing - 6 ft or more.

(2) Slightly fractured: fracture spacing - 2 to 6 ft.

(3) Moderately fractured: fracture spacing - 8 in. to 2 ft.

(4) Highly fractured: fracture spacing - 2 in. to 8 in.

(5) Intensely fractured: fracture spacing - 2 in. or less.

c. Dip of Bed or Fracture.

(1) Flat: 0 to 20 degrees.

(2) Dipping: 20 to 45 degrees.

(3) Steeply dipping: 45 to 90 degrees.

3. Discontinuities

a. Joints.

(1) Type: Type of joint if it can be readily determined (i.e., bedding, cleavage, foliation, schistosity, or extension).

(2) Degree of joint wall weathering:

(i) Unweathered: No visible signs are noted of weathering; joint wall rock is fresh, crystal bright.

(ii) Slightly weathered joints: Discontinuities are stained or discolored and may contain a thin coating of altered material. Discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20 percent of the discontinuity spacing.

(iii) Moderately weathered joints: Slight discoloration extends from discontinuity planes for greater than 20 percent of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.

(Continued)

Table 4-1. (Concluded)

(iv) Highly weathered joints: same as Item 1.a.(4).

(v) Completely weathered joints: same as Item 1.a.(5).

(3) Joint wall separations: General description of separation it it can be estimated from rock core; open or closed; if open note magnitude; filled or clean.

(4) Roughness:

(i) Very rough: Near vertical ridges occur on the discontinuity surface.

(ii) Rough: Some ridges are evident; asperities are clearly visible and discontinuity surface feels very abrasive.

(iii) Slighlty rough: Asperities on the discontinuity surface are distinguishable and can be felt.

(iv) Smooth: Surface appears smooth and feels so to the touch.

(v) Slickensided: Visual evidence of polishing exists.

(5) Infilling: Source, type, and thickness of infilling; alterated rock, or by deposition; clay, silt, etc.; how thick is the filler.

b. Faults and Shear Zones.

(1) Extent: Single plane or zone; how thick.

(2) Character: Crushed rock, gouge, clay infilling, slickensides.

b. Rock type. Rock type refers to the general geologic classification of the rock (e.g. basalt, sandstone, limestone, etc.). Certain physical characteristics are ascribed to a particular rock type with a geological name given according to the rocks mode of origin. Although the rock type is used primarily for identification and correlation, the type is often an important preliminary indicator of rock mass behavior.

c. Degree of weathering. The engineering properties of a rock can be, and often are, altered to varying degrees by weathering of the rock material. Weathering, which is disintegration and decomposition of the in-situ rock, is generally depth controlled, that is, the degree of weathering decreases with increasing depth below the surface.

d. Hardness. Hardness is a fundamental characteristic used for classification and correlation of geologic units. Hardness is an indicator of intact rock strength and deformability.

e. Texture. The strength of an intact rock is frequently affected, in part, by the individual grains comprising the rock.

f. Structure. Rock structure descriptions describe the frequency of discontinuity spacing and thickness of bedding. Rock mass strength and deformability are both influenced by the degree of fracturing.

g. Condition of discontinuities. Failure of a rock mass seldom occurs through intact rock but rather along discontinuities. The shear strength along a joint is dependent upon the joint aperture presence or absence of filling materials, the type of the filling material and roughness of the joint surface walls, and pore pressure conditions.

h. Color. The color of a rock type is used not only for identification and correlation, but also for an index of rock properties. Color may be indicative of the mineral constituents of the rock or of the type and degree of weathering that the rock has undergone.

i. Alteration. The rock may undergo alteration by geologic processes at depth, which is distinctively different from the weathering type of alteration near the surface.

4-5. Supplemental Descriptors

Descriptors and descriptor criterion discussed in paragraph 4-4 and summarized in Table 4-1 can be readily obtained from observation and inspection of rock core. However, certain important additional descriptors cannot be obtained from core alone. These additional descriptors include orientation of discontinuities, actual thicknesses of discontinuities, first-order roughness of discontinuities, continuity of discontinuities, cavity details, and slake durability.

a. Orientation of discontinuities. Because discontinuities represent directional planes of weakness, the orientation of the discontinuity is an important consideration in assessing sliding stability and, to some extent, bearing capacity and deformation/settlement. Retrieved core, oriented with respect to vertical and magnetic north, provides a means for determining discontinuity orientation. A number of manufacturers market devices for this purpose. However, most of these techniques abound with practical difficulties (e.g. see Hoek and Bray 1974). The sidewalls of the borehole from which conventional core has been extracted offer a unique picture of the subsurface where all structural features of the rock mass are still in their original position. In this respect, techniques that provide images of the borehole sidewalls such as the borehole camera, the borescope, TV camera or sonic imagery (discussed in Chapter 3, EM 1110-1-1804, EP 1110-1-10, and EM 1110-1-1802) offer an ideal means of determining the strike and dip angles of discontinuities. The orientation of the discontinuity should be recorded on a borehole photo log. The poles of the planes defined by the strike and dip angles of the discontinuities should then be plotted on an equal area stereonet. Equal area stereonet pole plots permit a statistical evaluation of discontinuity groupings or sets, thus establishing likely bounds of strike and dip orientations. A stereographic projection plot should then be made of the bounding discontinuity planes for each set of discontinuities to assess those planes which are kinematically free to slide. Goodman (1976), Hoek and Bray (1974), and Priest (1985) offer guidance for stereonet pole plots and stereographic projection techniques.

b. Discontinuity thickness. The drilling and retrieving of a rock core frequently disturb the discontinuity surfaces. For this reason, aperture measurements of discontinuity surfaces obtained from rock core can be misleading. The best source for joint aperture information is from direct measurement of borehole surface images (e.g. borehole photographs and TV camera recordings). The actual aperture measurement should be recorded on a borehole photo log. An alternative to recording actual measurements is to describe aperture according to the following descriptors:

(1) Very tight: separations of less than 0.1 mm.

(2) Tight: separations between 0.1 and 0.5 mm.

(3) Moderately open: separations between 0.5 and 2.5 mm.

(4) Open: separations between 2.5 and 10 mm.

(5) Very wide: separations between 10 and 25 mm.

For separations greater than 25 mm the discontinuity should be described as a major discontinuity.

c. First-order roughness of discontinuities. First-order roughness refers to the overall, or large scale, asperities along a discontinuity surface. Figure 4-1 illustrates the difference between first-order large scale asperities and the smaller, second-order asperities commonly associated with roughnesses representative of the rock core scale. The first-order roughness is generally the major contributor to shear strength development along a discontinuity (see paragraph 4-14*b* below for further discussion). A description of this large scale roughness can only be evaluated from an inspection of exposed discontinuity traces or surfaces. An inspection of rock outcrops in the vicinity of the project site offers an inexpensive means of obtaining this information. Critically oriented joint sets, for which outcrops are not available, may require excavation of inspection adits or trenches. Descriptors such as stepped, undulating, or planar should be used to describe noncritical surfaces. For critically oriented discontinuities, the angles of inclination, (referred to as the *i* angle) between the average dip of the discontinuity and first-order asperities should be measured and recorded

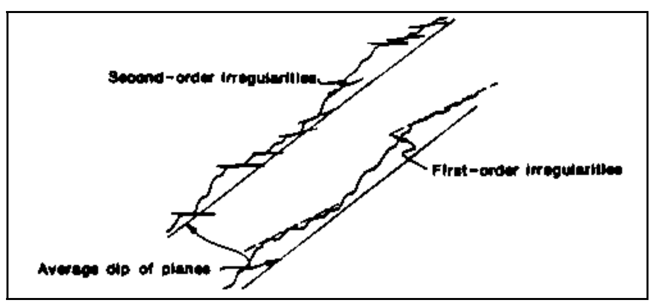

Figure 4-1. Rough discontinuity surface with first-order and second-order asperities (after Patton and Deere 1970)

(Figure 4-1). Hoek and Bray (1974) provide guidance for measuring first-order asperity angles.

d. Continuity of discontinuities. The continuity of a joint influences the extent to which the intact rock material and the discontinuities separately affect the behavior of the rock mass. In essence, the continuity, or lack of continuity, determines whether the strength that controls the stability of a given structure is representative of a discontinuous rock surface or a combination of discontinuous surfaces and intact rock. For the case of retaining structures, such as gravity dams and lockwalls, a discontinuity is considered fully continuous if its length is greater than the base width in the direction of potential sliding.

e. Cavities. Standard rock coring procedures are capable of detecting the presence of cavities as well as their extent along the borehole axis. However, an evaluation of the volumetric dimensions requires three-dimensional inspection. Downhole TV cameras, with their relatively long focal lengths, provide a means for inspecting cavities. Rock formations particularly susceptible to solutioning (e.g. karstic limestone, gypsum, and anhydrite) may require excavation of inspection trenches or adits to adequately define the location and extent of major cavities. A description of a cavity should include its geometric dimensions, the orientation of any elongated features, and the extent of any infilling as well as the type of infilling material.

4-6. Index Tests

Intact samples of rock may be selected for index testing to further aid in geological classification and as indicators of rock mass behavior. As a matter of routine, certain tests will always be performed on representative cores from each major lithological unit and/or weathered class. The number of tests should be sufficient to characterize the range of properties. Routine tests include water content, unit weight, and unconfined compression tests. Additional tests for durability, tensile strength, specific gravity, absorption, pulse velocity, and ultrasonic elastic constants and permeability tests as well as a petrographic examination may be dictated by the nature of the rock or by the project requirements. Types of classification and index tests which are frequently used for rock are listed in Table 4-2.

Section II
Rock Mass Classification

4-7. General

Following an appropriate amount of site investigation the rock mass can be divided or classified into zones or masses of similar expected performance. The similar performance may be excavatability, strength, deformability, or any other characteristic of interest, and is determined by use of all of the investigative tools previously described. A good rock mass classification system will:

- Divide a particular rock mass into groups of similar behavior.
- Provide a basis for understanding the characteristics of each group.
- Facilitate planning and design by yielding quantitative data required for the solution of real engineering problems.
- Provide a common basis for effective communication among all persons concerned with a given project.

A meaningful and useful rock mass classification system must be clear and concise, using widely accepted terminology. Only the most significant properties, based on measured parameters that can be derived quickly and inexpensively, should be included. The classification should be general enough that it can be used for a tunnel, slope, or foundation. Because each feature of a rock mass (i.e. discontinuities, intact rock, weathering, etc.) has a different significance, a ranking of combined factors is necessary to satisfactorily describe a rock mass. Each project may need site-specific zoning or rock mass classification, or it may benefit from use of one of the popular existing systems.

4-8. Available Classification Systems

Numerous rock mass classification systems have been developed for universal use. However, six have enjoyed greater use. The six systems include Terzaghi's Rock Load Height Classification (Terzaghi 1946); Lauffer's

Table 4-2
Laboratory Classification and Index Tests for Rock

Test	Test Method	Remarks
Unconfined (uniaxial) compression	RTH[1] 111	Primary index test for strength and deformability of intact rock; required input to rock mass classification systems.
Point load test	RTH 325	Indirect method to determine unconfined compressive (UC) strength; can be performed in the field on core pieces unsuitable for UC tests.
Water content	RTH 106	Indirect indication of porosity of intact rock or clay content of sedimentary rock.
Unit weight and total porosity	RTH 109	Indirect indication of weathering and soundness.
Splitting strength of rock (Brazilian tensile strength method)	RTH 113	Indirect method to determine the tensile strength of intact rock.
Durability	ASTM[2] D-4644	Index of weatherability of rock exposed in excavations.
Specific gravity of solids	RTH 108	Indirect indication of soundness of rock intended for use as riprap and drainage aggregate.
Pulse velocities and elastic constants	RTH 110	Index of compressional wave velocity and ultrasonic elastic constants for correlation with in-situ geophysical test results.
Rebound number	RTH 105	Index of relative hardness of intact rock cores.
Permeability	RTH 114	Intact rock (no joints or major defects).
Petrographic examination	RTH 102	Performed on representative cores of each significant lithologic unit.
Specific gravity and absorption	RTH 107	Indirect indication of soundness and deformability

Notes:
1. Rock Testing Handbook.
2. American Society for Testing and Materials.

Classification (Lauffer 1958); Deere's Rock Quality Designation (RQD) (Deere 1964); RSR Concept (Wickham, Tiedemann, and Skinner 1972); Geomechanics System (Bieniawski 1973); and the Q-System (Barton, Lien, and Lunde 1974). Most of the above systems were primarily developed for the design of underground excavations. However, three of the above six classification systems have been used extensively in correlation with parameters applicable to the design of rock foundations. These three classification systems are the Rock Quality Designation, Geomechanics System, and the Q-System.

4-9. Rock Quality Designation

Deere (1964) proposed a quantitative index obtained directly from measurements of rock core pieces. This index, referred to as the Rock Quality Designation (RQD), is defined as the ratio (in percent) of the total length of sound core pieces 4 in. (10.16 cm) in length or longer to the length of the core run. The RQD value, then, is a measure of the degree of fracturing, and, since the ratio counts only sound pieces of intact rock, weathering is accounted for indirectly. Deere (1964) proposed the following relationship between the RQD index and the engineering quality of the rock mass. The determination of RQD during core recovery is simple and straightforward. The RQD index is internationally recognized

RQD, percent	Rock Quality
< 25	Very poor
25 < 50	Poor
50 < 75	Fair
75 < 90	Good
90 < 100	Excellent

as an indicator of rock mass conditions and is a necessary input parameter for the Geomechanic System and Q-System. Since core logs should reflect to the maximum extent possible the rock mass conditions encountered, RQD should be determined in the field and recorded on the core logs. Deere and Deere (1989) provides the latest guidance for determining RQD.

4-10. Geomechanics Classification

a. General. The Geomechanics Classification, or Rock Mass Rating (RMR) system, proposed by Bieniawski (1973), was initially developed for tunnels. In recent years, it has been applied to the preliminary design of rock slopes and foundations as well as for estimating the in-situ modulus of deformation and rock mass strength. The RMR uses six parameters that are readily determined in the field:

- Uniaxial compressive strength of the intact rock.
- Rock Quality Designation (RQD).
- Spacing of discontinuities.
- Condition of discontinuities.
- Ground water conditions.
- Orientation of discontinuities.

All but the intact rock strength are normally determined in the standard geological investigations and are entered on an input data sheet (see Table B-1, Appendix B). The uniaxial compressive strength of rock is determined in accordance with standard laboratory procedures but can be readily estimated on site from the point-load strength index (see Table 4-2).

b. Basic RMR determination. The input data sheet (Table B-1, Appendix B) summarizes, for each core hole, all six input parameters. The first five parameters (i.e. strength, RQD, joint spacing, joint conditions, and ground water) are used to determine the basic RMR. Importance ratings are assigned to each of the five parameters in accordance with Part A of Table B-2, Appendix B. In assigning the rating for each core hole, the average conditions rather than the worst are considered. The importance ratings given for joint spacings apply to rock masses having three sets of joints. Consequently, a conservative assessment is obtained when only two sets of discontinuities are present. The basic rock mass rating is obtained by adding up the five parameters listed in Part A of Table B-2, Appendix B.

c. Adjustment for discontinuity orientation. Adjustment of the basic RMR value is required to include the effect of the strike and dip of discontinuities. The adjustment factor (a negative number) and hence the final RMR value, will vary depending upon the engineering application and the orientation of the structure with respect to the orientation of the discontinuities. The adjusted values, summarized in Part B of Table B-2, Appendix B, are divided into five groups according to orientations which range from very favorable to very unfavorable. The determination of the degree of favorability is made by reference to Table B-3 for assessment of discontinuity orientation in relation to dams (Part A), and tunnels (Part B).

d. Rock mass class. After the adjustment is made in accordance with Part B, Table B-2, Appendix B, the rock mass ratings are placed in one of five rock mass classes in Part C, Table B-2, Appendix B. Finally, the ratings are grouped in Part D of Table B-2, Appendix B. This section gives the practical meaning of each rock class, and a qualitative description is provided for each of the five rock mass classes. These descriptions range from “very good rock” for class I (RMR range from 81 to 100) to “very poor rock” for class V (RMR ranges < 20). This classification also provides a range of cohesion values and friction angles for the rock mass.

4-11. Q-System

The Q-system, proposed by Barton, Lien, and Lunde (1974) was developed specifically for the design of tunnel support systems. As in the case of the Geomechanics System, the Q-system has been expanded to provide preliminary estimates. Likewise, the Q-system incorporates the following six parameters and the equation for obtaining rock mass quality Q:

- Rock Quality Designation (RQD).
- Number of discontinuity sets.
- Roughness of the most unfavorable discontinuity.
- Degree of alteration or filling along the weakest discontinuity.
- Water inflow.
- Stress condition.

$$Q = (\mathrm{RQD}/J_n) \times (J_r/J_a) \times (J_w/\mathrm{SRF}) \qquad (4\text{-}1)$$

where

RQD = Rock Quality Designation

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction number

SRF = stress reduction number

Table B-4, Appendix B, provides the necessary guidance for assigning values to the six parameters. Depending on the six assigned parameter values reflecting the rock mass quality, Q can vary between 0.001 to 1000. Rock quality is divided into nine classes ranging from exceptionally poor (Q ranging from 0.001 to 0.01) to exceptionally good (Q ranging from 400 to 1000).

4-12. Value of Classification Systems

There is perhaps no engineering discipline that relies more heavily on engineering judgment than rock mechanics. This judgment factor is, in part, due to the difficulty in testing specimens of sufficient scale to be representative of rock mass behavior and, in part, due to the natural variability of rock masses. In this respect, the real value of a rock mass classification systems is appropriately summarized by Bieniawski (1979). "...no matter which classification system is used, the very process of rock mass classification enables the designer to gain a better understanding of the influence of the various geologic parameters in the overall rock mass behavior and, hence, gain a better appreciation of all the factors involved in the engineering problem. This leads to better engineering judgment. Consequently, it does not really matter that there is no general agreement on which rock classification system is best; it is better to try two or more systems and, through a parametric study, obtain a better "feel" for the rock mass. Rock mass classification systems do not replace site investigations, material descriptions, and geologic work-up. They are an adjunct to these items and the universal schemes, in particular, have special value in relating the rock mass in question to engineering parameters based on empirical knowledge."

Section III
Shear Strength

4-13. General

The shear strength that can be developed to resist sliding in a rock foundation or a rock slope is generally controlled by natural planes of discontinuity rather than the intact rock strength. The possible exception to this rule may include structures founded on, or slopes excavated in, weak rock or where a potential failure surface is defined by planes of discontinuities interrupted by segments of intact rock blocks. Regardless of the mode of potential failure, the selection of shear strength parameters for use in the design process invariably involves the testing of appropriate rock specimens. Selection of the type of test best suited for intact or discontinuous rock, as well as selection of design shear strength parameters, requires an appreciation of rock failure characteristics. Discussions on rock failure characteristics are contained in TR GL-83-13 (Nicholson 1983a) and Goodman (1980).

4-14. Rock Failure Characteristics

Failure of a foundation or slope can occur through the intact rock, along discontinuities or through filling material contained between discontinuities. Each mode of failure is defined by its own failure characteristics.

a. Intact rock. At stress levels associated with low head gravity dams, retaining walls and slopes, virtually all rocks behave in a brittle manner at failure. Brittle failure is marked by a rapid increase in applied stress, with small strains, until a peak stress is obtained. Further increases in strain cause a rapid decrease in stress until the residual stress value is reached. While the residual stress value is generally unique for a given rock type and minor principal stress, the peak stress is dependent upon the size of the specimen and the rate that the stress is applied. Failure envelopes developed from plots of shear stress versus normal stress are typically curvilinear.

b. Discontinuities. The typical failure envelope for a clean discontinuous rock is curvilinear as is intact rock. Surfaces of discontinuous rock are composed of irregularities or asperities ranging in roughness from almost smooth to sharply inclined peaks. Conceptually there are three modes of failure--asperity override at low normal stresses, failure through asperities at high normal stresses, and a combination of asperity override and failure through asperities at intermediate normal stresses. Typically, those normal stresses imposed by Corps structures are sufficiently low that the mode of failure will be controlled by asperity override. The shear strength that can be developed for the override mode is scale dependent. Initiation of shear displacement causes the override mode to shift from the small scale second-order irregularities to the large scale first-order irregularities. As indicated in Figure 4-1, first-order irregularities generally have smaller angles of inclination (*i* angles) than second-order irregularities. Shear strengths of discontinuities with rough undulating surfaces reflect the largest scale effects with small surface areas (laboratory specimen size) developing higher shear stress than large surface areas (in-situ scale). Figure 4-2 illustrates the influence of both scale effects and discontinuity surface roughnesses.

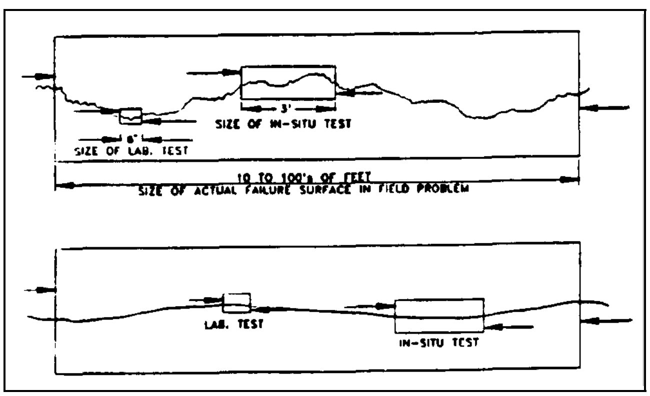

Figure 4-2. Effect of different size specimens selected along a rough and a smooth discontinuity surface (after Deere et al. 1967)

c. Filled discontinuities. Failure modes of filled discontinuities can range from those modes associated with clean unfilled discontinuities to those associated with soil. Four factors contribute to their strength behavior: thickness of the filler material, material type, stress history and displacement history.

(1) Thickness. Research indicates that the strength of discontinuities with filler thicknesses greater than two times the amplitude of the surface undulations is controlled by the strength of the filler material. In general, the thicker the filler material with respect to the amplitude of the asperities, the less the scale effects.

(2) Material type. The origin of the filler material and the strength characteristics of the joint are important indicators. Sources of filler material include products of weathering or overburden washed into open, water-conducting discontinuities; precipitation of minerals from the ground water; by-products of weathering and alterations along joint walls; crushing of parent rock surfaces due to tectonic and shear displacements; and thin seams deposited during formation. In general, fine-grained clays are more frequently found as fillers and are more troublesome in terms of structural stability.

(3) Stress history. For discontinuities containing fine-grained fillers, the past stress history determines whether the filler behaves as a normally consolidated or overconsolidated soil.

(4) Displacement history. An important consideration in determining the strength of discontinuities filled with fine-grained cohesive materials is whether or not the discontinuity has been subjected to recent displacement. If significant displacement has occurred, it makes little difference whether the material is normally or overconsolidated since it will be at or near its residual strength.

4-15. Failure Criteria

a. Definition of failure. The term “failure” as applied to shear strength may be described in terms of load, stress, deformation, strain or other parameters. The failure strengths typically associated with the assessment of sliding stability are generally expressed in terms of peak, residual, ultimate or as the shear strength at a limiting strain or displacement as illustrated in Figure 4-3.

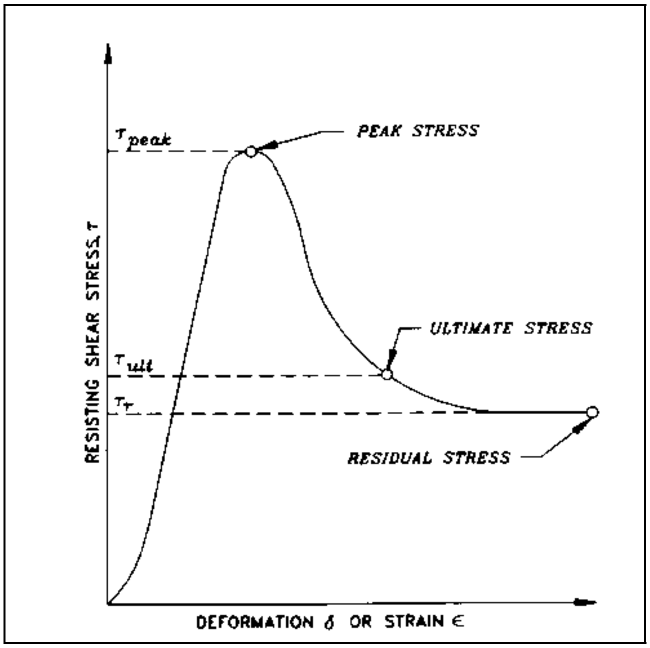

Figure 4-3. Shear test failure as defined by peak, ultimate, and residual stress levels (after Nicholson 1983a)

The appropriate definition of failure generally depends on the shape of the shear stress versus shear deformation/strain curve as well as the mode of potential failure. Figure 4-4 illustrates the three general shear stress versus deformation curves commonly associated with rock failure.

b. Linear criteria. Failure criteria provide an algebraic expression for relating the shear strength at failure with a mathematical model necessary for stability analysis. Mathematical limit equilibrium models used to access sliding stability incorporate linear Mohr-Coulomb failure criterion as follows:

$$\tau_f = c + \sigma_n \tan \phi \tag{4-2}$$

where

τ_f = the shearing stress developed at failure

σ_n = stress normal to the failure plane

The c and ϕ parameters are the cohesion intercept and angle of internal friction, respectively. Figure 4-5 illustrates the criterion. It must be recognized that failure envelopes developed from shear tests on rock are generally curved. However, with proper interpretation, failure envelopes over most design stress ranges can be closely approximated by the linear Coulomb equation required by the analytical stability model.

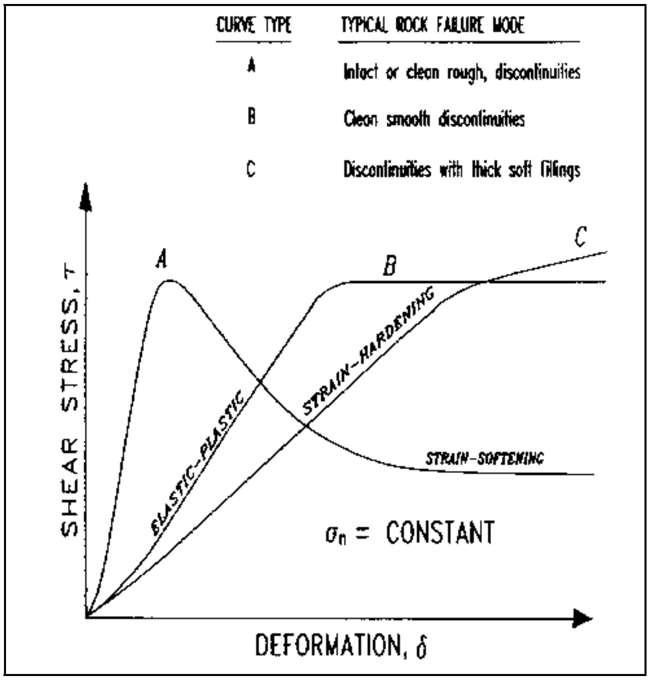

Figure 4-4. Hypothetical shear stress-deformation curves from drained direct shear tests on: (a) strain-softening; (b) elastic-plastic; and (c) strain-hardening materials (after Nicholson 1983a)

c. Bilinear criteria. Bilinear criteria (Patton 1966; Goodman 1980) offer a more realistic representation of the shear stress that can be developed along clean (unfilled) discontinuities. These criteria divide a typical curved envelope into two linear segments as illustrated in Figure 4-6. The maximum shear strength that can be developed at failure is approximated by the following equations:

$$\tau_f = \sigma_n \tan (\phi_u + i) \tag{4-3}$$

and

$$\tau_f = c_a + \sigma_n \tan \phi_r \tag{4-4}$$

where

τ_f = maximum (peak) shear strength at failure

σ_n = stress normal to the shear plane (discontinuity)

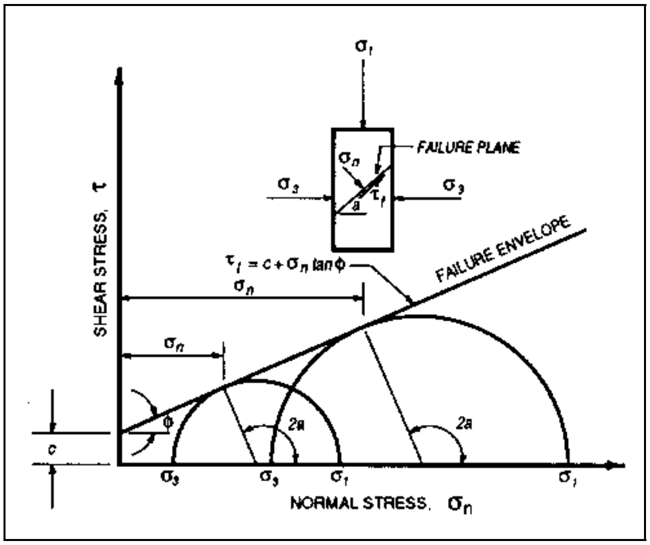

Figure 4-5. Mohr-Coulomb relationship with respect to principal stresses and shear stress

ϕ_u = the basic friction angle on smooth planar sliding surface

i = angle of inclination of the first order (major) asperities

ϕ_r = the residual friction angle of the material comprising the asperities

c_a = the apparent cohesion (shear strength intercept) derived from the asperities

For unweathered discontinuity surfaces, the basic friction angle and the residual friction angle are, for practical purposes, the same. The intercept of the two equations (i.e. σ_τ in Figure 4-6) occurs at the transition stress between the modes of failure represented by asperity override and shearing of the asperities. Normal stresses imposed by Corps projects are below the transition stress (σ_τ) for the majority of rock conditions encountered. Hence, maximum shear strengths predicted by Equation 4-3, generally control design.

4-16. Shear Strength Tests

Table 4-3 lists tests that are useful for measuring the shear strength of rock. Details of the tests, test apparatus, and procedures are given in the Rock Testing Handbook (see references Table 4-3), EM 1110-1-1804, and GL-83-14 (Nicholson 1983b.).

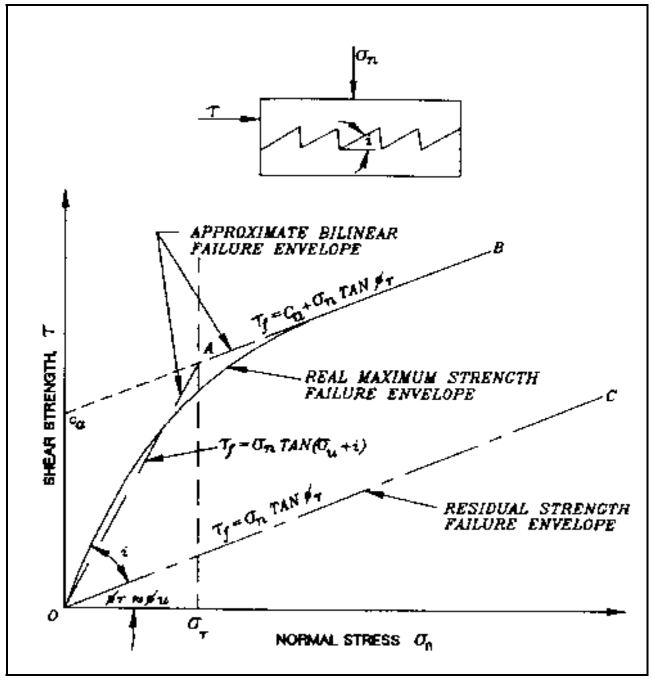

Figure 4-6. Typical approximate bilinear and real curvilinear failure envelopes for modeled discontinuous rock

4-17. Shear Strength Testing Program

The testing program for measuring shear strengths of rock specimens reflects the intended use of the test results (preliminary or final design), the type of specimens (intact or discontinuous), the cost, and, in some cases, the availability of testing devices. In general, the testing program closely parallels the field exploration program, advancing from preliminary testing where modes of potential failure are poorly defined to detailed testing of specific modes of potential failure controlling project design. As a minimum, the following factors should be considered prior to initiating the final detailed phase of testing: the sensitivity of stability with respect to strengths, loading conditions, suitability of tests used to model modes of failure, and the selection of appropriate test specimens.

a. Sensitivity. A sensitivity analysis should be performed to evaluate the relative sensitivity of the shear strengths required to provide an adequate calculated factor of safety along potential failure planes. Such analysis frequently indicates that conservative and inexpensively obtained strengths often provide an adequate measure of stability, without the extra cost of more precisely defined in-situ strengths.

Table 4-3
Tests to Measure Shear Strength of Rock

Test	Reference	Remarks
Laboratory direct shear	RTH 203[1]	Strength along planes of weakness (bedding), discontinuities or rock-concrete contact; not recommended for intact rock.
Laboratory triaxial	RTH 202	Deformation and strength of inclined compression planes of weakness and discontinuities; strain and strength of intact rock.
In-situ direct shear	RTH 321	Expensive; generally reserved for critically located discontinuities filled with a thin seam of very weak material.
In-situ uniaxial	RTH 324	Expensive; primarily used for defining compression scale effects of weak intact rock; several specimen sizes usually tested.

Notes:
1. Rock Testing Handbook.

b. Loading conditions. Shear tests on rock specimens should duplicate the anticipated range of normal stresses imposed by the project structure along potential failure planes. Duplication of the normal stress range is particularly important for tests on intact rock, or rough natural discontinuities, that exhibit strong curvilinear failure envelopes.

c. Shear test versus mode of failure. Both triaxial and direct shear tests are capable of providing shear strength results for all potential modes of failure. However, a particular type of test may be considered better suited for modes of failure. The suitability of test types with respect to modes of failure should be considered in specifying a testing program.

(1) Laboratory triaxial test. The triaxial compression test is primarily used to measure the undrained shear strength and in some cases the elastic properties of intact rock samples subjected to various confining pressures. By orienting planes of weakness the strength of natural joints, seams, and bedding planes can also be measured. The oriented plane variation is particularly useful for obtaining strength information on thinly filled discontinuities containing soft material. Confining pressures tend to prevent soft fillers from squeezing out of the discontinuity. The primary disadvantage of the triaxial test is that stresses normal to the failure plane cannot be directly controlled. Since clean discontinuities are free draining, tests on clean discontinuities are considered to be drained. Tests on discontinuities filled with fine-grained materials are generally considered to be undrained (drained tests are possible but require special testing procedures). Tests on discontinuities with coarse grained fillers are generally considered to be drained. Detailed procedures for making laboratory triaxial tests are presented in the Rock Testing Handbook (RTH 204).

(2) Laboratory direct shear test. The laboratory direct shear test is primarily used to measure the shear strength, at various normal stresses, along planes of discontinuity or weakness. Although sometimes used to test intact rock, the potential for developing adverse stress concentrations and the effects from shear box induced moments makes the direct shear test less than ideally suited for testing intact specimens. Specimen drainage conditions, depending on mode of failure, are essentially the same as for laboratory triaxial tests discussed above. The test is performed on core samples ranging from 2 to 6 inches in diameter. Detailed test procedures are presented in the Rock Testing Handbook (RTH 203).

(3) In-situ direct shear test. In-situ direct shear tests are expensive and are only performed where critically located, thin, weak, continuous seams exist within relatively strong adjacent rock. In such cases, conservative lower bound estimates of shear strength seldom provide adequate assurance against instability. The relatively

large surface area tested is an attempt to address unknown scale effects. However, the question of how large a specimen is large enough still remains. The test, as performed on thin, fine-grained, clay seams, is considered to be an undrained test. Test procedure details are provided in the Rock Testing Handbook (RTH 321). Technical Report S-72-12 (Zeigler 1972) provides an indepth review of the in-situ direct shear test.

(4) In-situ uniaxial compression test. In-situ uniaxial compression tests are expensive. The test is used to measure the elastic properties and compressive strength of large volumes of virtually intact rock in an unconfined state of stress. The uniaxial strength obtained is useful in evaluating the effects of scale. However, the test is seldom performed just to evaluate scale effects on strength.

d. Selection of appropriate specimens. No other aspect of rock strength testing is more important than the selection of the test specimens that best represents the potential failure surfaces of a foundation. Engineering property tests conducted on appropriate specimens directly influence the analysis and design of projects. As a project progresses, team work between project field personnel and laboratory personnel is crucial in changing type of test, test specimen type, and number of tests when site conditions dictate. The test specimen should be grouped into rock types and subgrouped by unconfined compressive strength, hardness, texture, and structure, or any other distinguishing features of the samples. This process will help in defining a material's physical and mechanical properties. General guidance on sample selection is provided in EM 1110-1-1804. However, shear strength is highly dependent upon the mode of failure, i.e. intact rock, clean discontinuous rock, and discontinuities containing fillers. Furthermore, it must be realized that each mode of failure is scale dependent. In this respect, the selection of appropriate test specimens is central to the process of selecting design shear strength parameters.

4-18. Selection of Design Shear Strength Parameters

a. Evaluation procedures. The rock mass within a particular site is subject to variations in lithology, geologic structure, and the in-situ stress. Regardless of attempts to sample and test specimens with flaws and/or weaknesses present in the rock mass, these attempts, at best, fall short of the goal. The number, orientation, and size relationship of the discontinuities and/or weaknesses may vary considerably, thus affecting load distribution and the final results. In addition to these factors, laboratory results are dependent on the details of the testing procedures, equipment, sampling procedures, and the condition of the sample at the time of the test. The result of these numerous variables is an expected variation in the laboratory test values which further complicates the problem of data evaluation. The conversion from laboratory measured strength parameters to in-situ strength parameters requires a careful evaluation and analysis of the geologic and laboratory test data. Also, a combination of experience and judgment is necessary to assess the degree or level of confidence that is required in the selected parameters. As a minimum, the following should be considered: the most likely mode of prototype failure, the factor of safety, the design use, the cost of tests, and the consequence of a failure. A flow diagram illustrating examples of factors to consider in assessing the level of confidence in selected design strengths is shown in Figure 4-7. In general, an increase in assessed confidence should either reflect increasing efforts to more closely define prototype shear strength, at increasing cost, or increasing conservatism in selected design strengths to account for the uncertainties of the in-situ strength.

b. Selection procedures. Failure envelopes for likely upper and lower bounds of shear strength can generally be determined for the three potential modes of failure; intact rock, clean discontinuities, and filled discontinuities. These limits bound the range within which the in-situ strength is likely to lie. Technical Report GL-83-13 (Nicholson 1983a) describes appropriate test methods and procedures to more accurately estimate in-situ strength parameters. Efforts to more accurately define in-situ strengths must reflect the level of confidence that is required by the design.

(1) Intact rock. Plots of shear stress versus normal stress, from shear test on intact rock, generally result in considerable data scatter. In this respect, nine or more tests are usually required to define both the upper and lower bounds of shear strength. Figure 4-8 shows a plot of shear stress versus normal stress for a series of tests on a weak limestone. Failure envelopes obtained from a least-squares best fit of upper and lower bounds, as well as all data points, are shown in Figure 4-8. Variations in cohesion values are generally greater than the variations in the friction angle values. With a sufficient number of tests to define scatter trends, over a given range of normal stresses the confidence that can be placed in the friction angle value exceeds the level of confidence that can be placed in the cohesion value. As a rule, a sufficient factor of safety can be obtained from lower bound

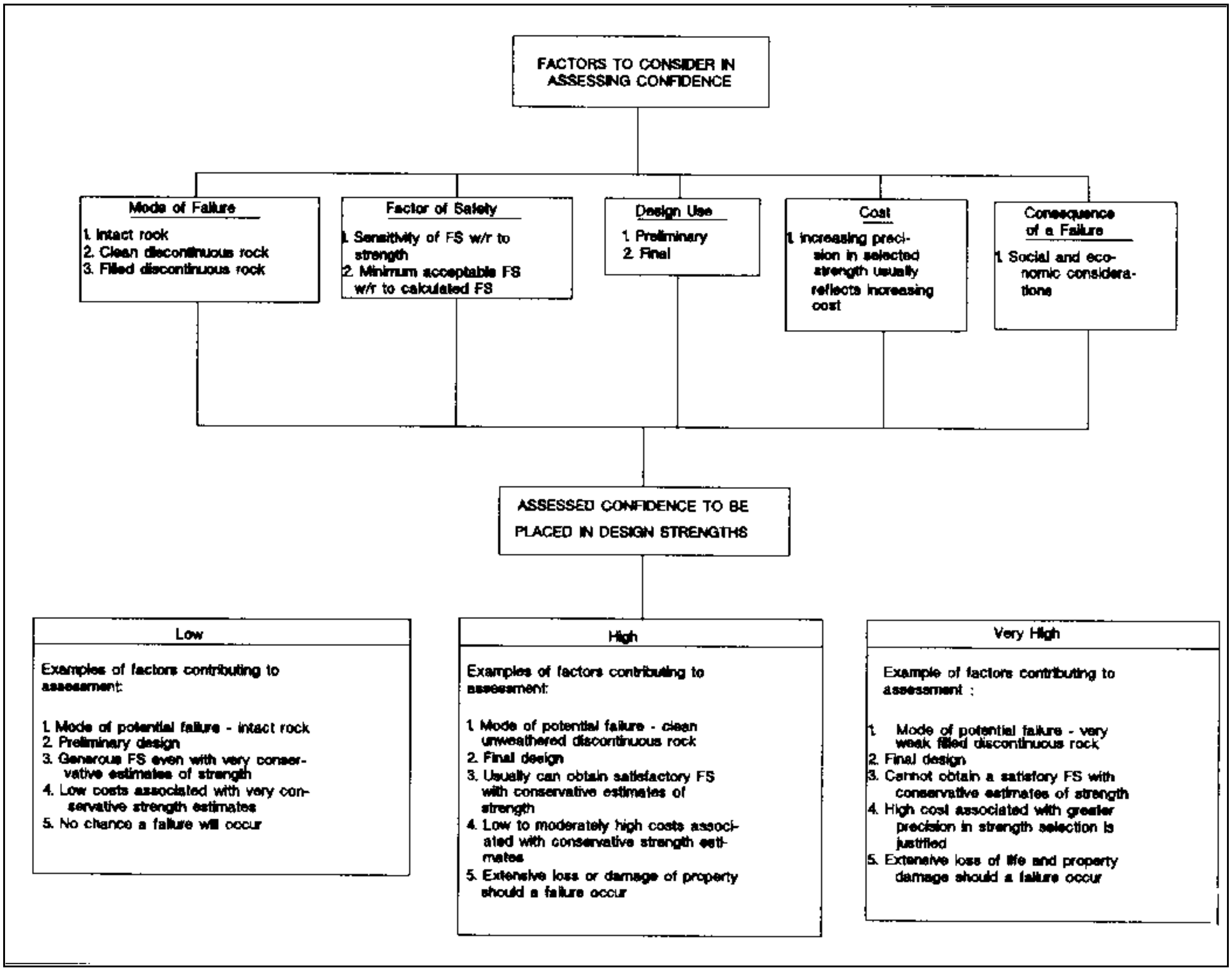

Figure 4-7. Flow diagram illustrating examples of factors to consider in assessing the confidence to be placed in selected design strengths (after Nicholson 1983a)

estimates of shear strength obtained from laboratory tests. For design cases where lower bound shear strength estimates provide marginal factors of safety, the influence of scale effects must be evaluated. Shear strengths obtained from laboratory tests on small specimens should be reduced to account for scale effects. In this respect, Pratt et al. (1972) and Hoek and Brown (1980) suggest that the full- scale uniaxial compressive strength of intact rock can be as much as 50 percent lower than the uniaxial compressive strength of a small intact laboratory specimen. In the absence of large scale tests to verify the effects of scale, conservative estimates of the shear strength parameters (cohesion and friction angle) which account for scale effects can be obtained by reducing the lower bound cohesion value by 50 percent. This reduced lower bound cohesion value is to be used with the lower bound friction angle value for marginal design cases.

(2) Clean discontinuities. Upper and lower bounds of shear strength for clean discontinuities can be obtained from laboratory tests on specimens containing natural discontinuities and presawn shear surfaces, respectively. The number of tests required to determine the bounds of strength depends upon the extent of data scatter observed in plots of shear stress versus normal stress. As a rule, rough natural discontinuity surfaces will generate more data scatter than smooth discontinuity surfaces. Hence, lower bound strengths obtained from tests on smooth sawn surfaces may require as few as three tests while upper bound strength from tests on very rough natural

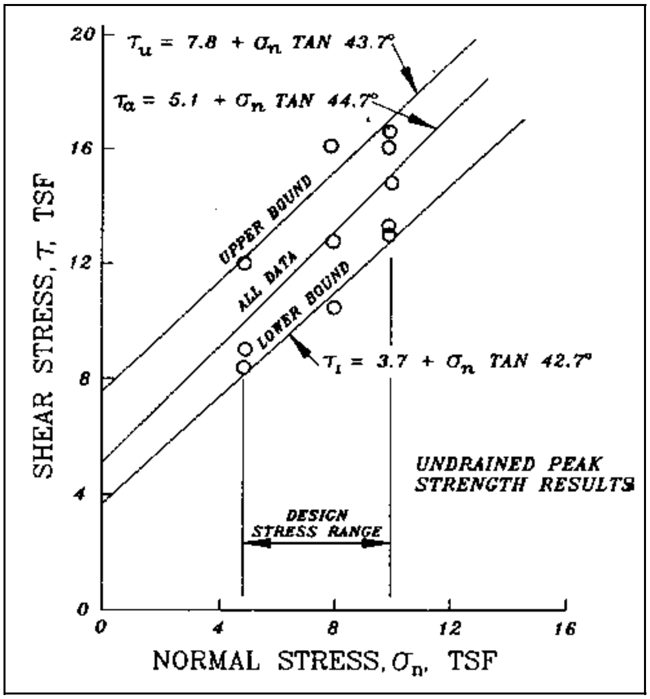

Figure 4-8. Direct shear test results on intact limestone illustrating upper and lower bounds of data scatter

engineering judgment can not be overly emphasized. discontinuity surfaces may require nine or more tests. Data scatter and/or curvilinear trends in plots of shear stress versus normal stress may result in cohesion intercepts. In such cases, cohesion intercepts are ignored in the selection of design shear strengths. The lower bound failure envelope obtained from shear tests on smooth sawn surfaces defines the basic friction angle (ϕ_u in Equation 4-3). The friction angle selected for design may be obtained from the sum of the basic friction angle and an angle representative of the effective angle of inclination (*i* in Equation 4-3) for the first-order asperities. The sum of the two angles must not exceed the friction angle obtained from the upper bound shear tests on natural discontinuities. The primary difficulty in selecting design friction values lies in the selection of an appropriate *i* angle. Discontinuity surfaces or outcrop traces of discontinuities are not frequently available from which to base a reasonable estimate of first order inclination angles. In such cases estimates of the *i* angle must rely on sound engineering judgment and extensive experience in similar geology.

(3) Filled discontinuities. In view of the wide variety of filler materials, previous stress and displacement histories and discontinuity thicknesses, standardization of a procedure for selecting design shear strengths representative of filled discontinuities is difficult. The process is further complicated by the difficulty in retrieving quality specimens that are representative of the discontinuity in question. For these reasons, the use of sound Uncertainties associated with unknown conditions effecting shear strength must be reflected in increased conservatism. Generally, the scale effects associated with discontinuous rock are lessened as the filler material becomes thicker in relation to the amplitude of the first-order joint surface undulations. However, potential contributions of the first-order asperities to the shear strength of a filled joint are, as a rule, not considered in the strength selection process because of the difficulty in assessing their effects. Shear strengths that are selected based on in-situ direct shear test of critically located weak discontinuities are the exception to this general rule, but there still remains the problem of appropriate specimen size. As illustrated in Figure 4-9, the displacement history of the discontinuity is of primary concern. If a filled discontinuity has experienced recent displacement, as evident by the presence of slicken-sides, gouge, mismatched joint surfaces, or other features, the strength representative of the joint is at or near its residual value. In such cases, shear strength selection should be based on laboratory residual shear tests of the natural joint. Possible cohesion intercepts observed from the test results should not be included in the selection of design strengths. If the discontinuity has not experienced previous displacement, the shear strength is at or near its peak value. Therefore, whether the filler material is normally or overconsolidated is of considerable importance. In this respect, the shear stress level used to define failure of laboratory test specimens is dependent upon the material properties of the filler. The following definitions of failure stress are offered as general guidance to be tempered with sound engineering judgment: peak strength should be used for filler consisting of normally consolidated cohesive materials and all cohesionless materials; peak or ultimate strength is used for filler consisting of overconsolidated cohesive material of low plasticity; ultimate strength, peak strength of remolded filler, or residual strength is used (depending on material characteristics) for filler consisting of overconsolidated cohesive material of medium to high plasticity.

(4) Combined modes. Combined modes of failure refer to those modes in which the critical failure path is defined by segments of both discontinuous planes and planes passing through intact rock. Selection of appropriate shear strengths for this mode of failure is

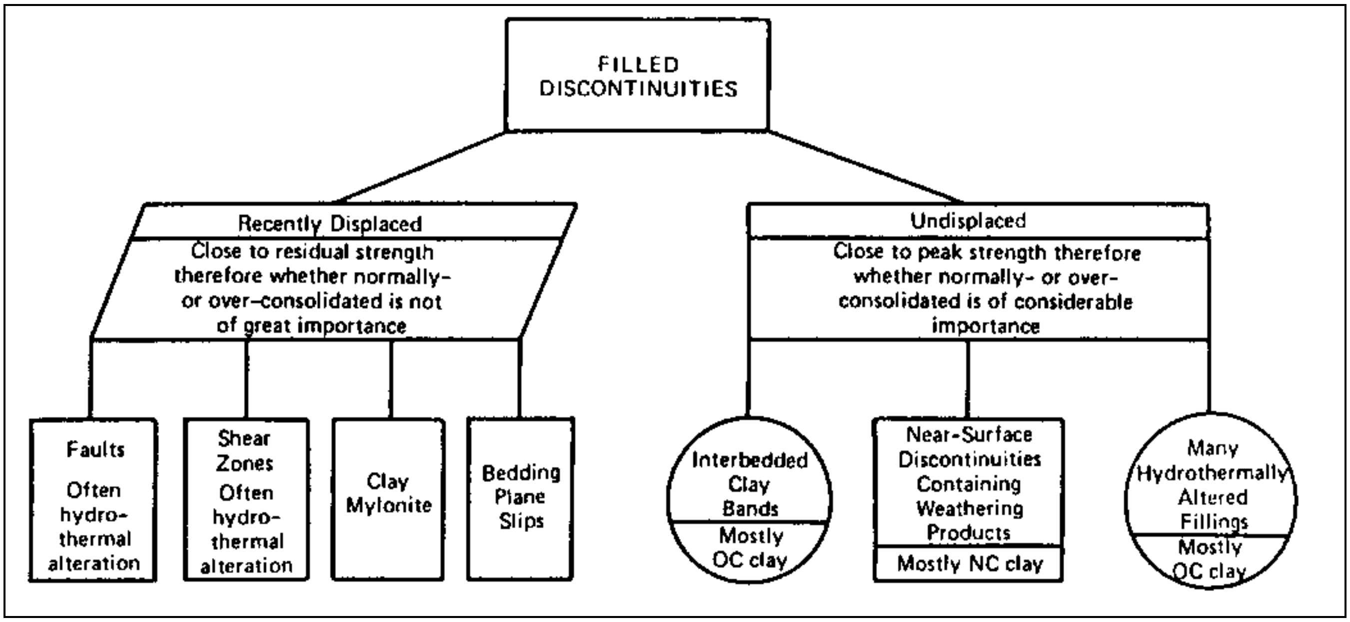

Figure 4-9. Simplified division of filled discontinuities into displaced and undisplaced and normally consolidated (NC) and overconsolidated (OC) categories (after Barton 1974)

particularly difficult for two reasons. First, the precentages of the failure path defined by discontinuities or intact rock are seldom known. Second, strains/displacements necessary to cause failure of intact rock are typically an order of magnitude (a factor of 10) smaller than those displacements associated with discontinuous rock. Hence, peak strengths of the intact rock proportion will already have been mobilized and will likely be approaching their residual strength before peak strengths along the discontinuities can be mobilized. For these reasons, selection of appropriate strengths must be based on sound engineering judgment and experience gained from similar projects constructed in similar geological conditions. Shear strength parameters selected for design must reflect the uncertainties associated with rock mass conditions along potential failure paths as well as mechanisms of potential failure (i.e. sliding along discontinuities versus shear through intact rock).

Section IV
Deformation and Settlement

4-19. General

The deformational response of a rock mass is important in seismic analyses of dams and other large structures as well as the static design of gravity and arch dams, tunnels, and certain military projects. Analytical solutions for deformation and settlement of rock foundations are invariably based on the assumption that the rock mass behaves as a continuum. As such, analytical methods used to compute deformations and the resulting settlements are founded on the theory of elasticity. The selection of design parameters, therefore, involves the selection of appropriate elastic properties: Poisson's ratio and the elastic modulus. Although it is generally recognized that the Poisson's ratio for a rock mass is scale and stress dependent, a unique value is frequently assumed. For most rock masses, Poisson's ratio is between 0.10 and 0.35. As a rule, a poorer quality rock mass has a lower Poisson's ratio than good quality rock. Hence, the Poisson's ratio for a highly fractured rock mass may be assumed as 0.15 while the value for a rock mass with essentially no fractures may be assumed as equal to the value of intact rock. A method for determining Poisson's ratios for intact rock core specimens is described in the Rock Testing Handbook (RTH 201). The selection of an appropriate elastic modulus is the most important parameter in reliable analytical predictions of deformation and settlement. Rock masses seldom behave as an ideal elastic material. Furthermore, modulus is both scale and stress dependent. As a result, stress-strain responses typical of a rock mass are not linear. The remaining parts of this section will address appropriate definitions of modulus, scale effects, available methods for estimating modulus values and the selection of design values.

4-20. Moduli Definitions

The elastic modulus relates the change in applied stress to the change in the resulting strain. Mathematically, it is expressed as the slope of a given stress-strain response. Since a rock mass seldom behaves as an ideal linear elastic material, the modulus value is dependent upon the proportion of the stress-strain response considered. Figure 4-10 shows a stress-strain curve typical of an in-situ rock mass containing discontinuities with the various moduli that can be obtained. Although the curve, as shown, is representative of a jointed mass, the curve is also typical of intact rock except that upper part of the curve tends to be concaved downward at stress levels approaching failure. As can be seen in Figure 4-10 there are at least four portions of the stress-strain curve used for determining in-situ rock mass moduli: the initial tangent modulus, the elastic modulus, the tangent recovery modulus, and the modulus of deformation.

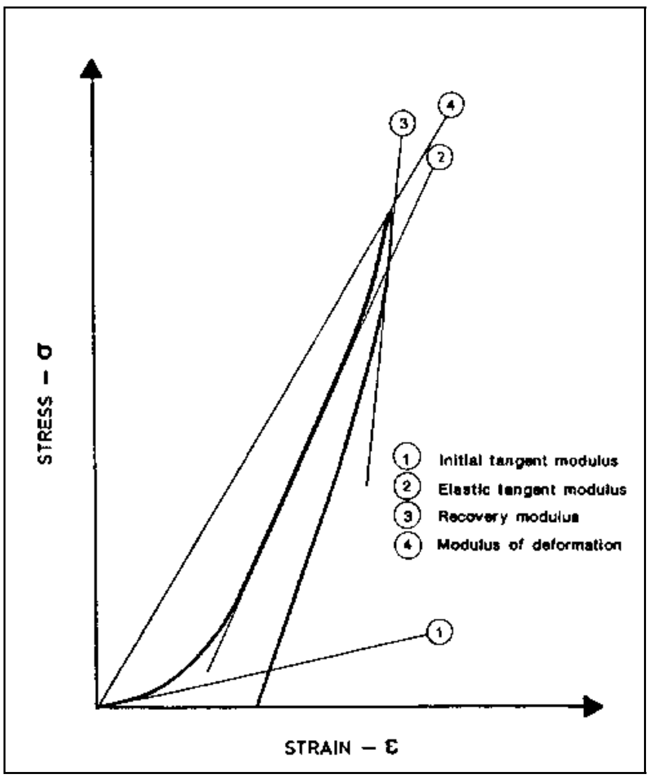

Figure 4-10. Stress-strain curve typical of in-situ rock mass with various moduli that can be obtained

a. Initial tangent modulus. The initial tangent modulus is determined from the slope of a line constructed tangent to the initial concave upward section of the stress-strain curve (i.e. line 1 in Figure 4-10). The initial curved section reflects the effects of discontinuity closure in in-situ tests and micro-crack closure in tests on small laboratory specimens.

b. Elastic modulus. Upon closure of discontinuities/ micro-cracks, the stress-strain becomes essentially linear. The elastic modulus, frequently referred to as the modulus of elasticity, is derived from the slope of this linear (or near linear) portion of the curve (i.e. line 2 in Figure 4-10). In some cases, the elastic modulus is derived from the slope of a line constructed tangent to the stress-strain curve at some specified stress level. The stress level is usually specified as 50 percent of the maximum or peak stress.

c. Recovery modulus. The recovery modulus is obtained from the slope of a line constructed tangent to the initial segment of the unloading stress-strain curve (i.e. line 3 in Figure 4-10). As such, the recovery modulus is primarily derived from in-situ tests where test specimens are seldom stressed to failure.

d. Modulus of deformation. Each of the above moduli is confined to specific regions of the stress-strain curve. The modulus of deformation is determined from the slope of the secant line established between zero and some specified stress level (i.e. line 4 in Figure 4-10). The stress level is usually specified as the maximum or peak stress.

4-21. Test Methods for Estimating Modulus

There are at least nine different test methods available to estimate rock modulus. While all nine methods have been used in estimating modulus for design purpose, only the following seven have been standardized: the uniaxial compression tests; uniaxial-jacking tests; the pressure-meter test; plate load test; pressure-chamber tests; radial-jack tests; and borehole-jacking tests. Other test methods that are not standardized but are described in the literature include flat-jack tests and tunnel-relaxation tests.

a. Uniaxial compression tests. Laboratory uniaxial compression tests are the most frequently used tests for estimating rock modulus. These tests are performed on relatively small, intact, specimens devoid of discontinuities. As such, the results obtained from these tests over estimate the modulus values required for design analyses. Laboratory tests are useful in that the derived moduli provide an upper limit estimate. In-situ uniaxial compression tests are capable of testing specimens of sufficient size to contain a representative number of discontinuities. Modulus values obtained from in-situ tests are considered to be more reliable. This test method is more versatile than some in-situ methods in that test specimens can be developed from any exposed surface. However, the tests are expensive. The Rock Testing Handbook describes test procedures for both laboratory (RTH 201) and in-situ (RTH 324) uniaxial compression tests for the estimation of modulus.

b. Uniaxial jacking tests. The uniaxial jack test involves the controlled loading and unloading of opposing rock surfaces developed in a test adit or trench. The loads are applied by means of large hydraulic jacks which react against two opposing bearing pads. Measurement of the rock mass deformational response below the bearing pads provides two sets of data from which moduli can be derived. The test is expensive. However, the majority of the expense is associated with the excavation of the necessary test adit or trench. The test procedures are described in the Rock Testing Handbook (RTH 365).

c. Pressure meter tests. The pressure meter test expands a fluid filled flexible membrane in a borehole causing the surrounding wall of rock to deform. The fluid pressure and the volume of fluid equivalent to the volume of displaced rock are recorded. From the theory of elasticity, pressure and volume changes are related to the modulus. The primary advantage of the pressure meter is its low cost. The test is restricted to relatively soft rock. Furthermore, the test influences only a relatively small volume of rock. Hence, modulus values derived from the tests are not considered to be representative of rock mass conditions. The test procedures are described in the Rock Testing Handbook (RTH 362).

d. Plate load tests. The plate load test is essentially the same as the uniaxial jacking test except that only one surface is generally monitored for deformation. If sufficient reaction such as grouted cables can be provided, the test may be performed on any rock surface. Details of the test procedures are discussed in the Rock Testing Handbook (RTH 364-89).

e. Flat-jack tests. The flat-jack test is a simple test in which flat-jacks are inserted into a slot cut into a rock surface. Deformation of the rock mass caused by pressurizing the flat-jack is measured by the volumetric change in the jack fluid. The modulus is derived from relationships between jack pressure and deformation. However, analysis of the test results is complicated by boundary conditions imposed by the test configuration. The primary advantages of the test lie in its ability to load a large volume of rock and its relatively low cost. The test procedures are described by Lama and Vutukuri (1978).

f. Pressure-chamber tests. Pressure-chamber tests are performed in large, underground openings. Generally, these openings are test excavations such as exploratory tunnels or adits. Pre-existing openings, such as caves or mine chambers, can be used if available and applicable to project conditions. The opening is lined with an impermeable membrane and subjected to hydraulic pressure. Instrumented diametrical gages are used to record changes in tunnel diameter as the pressure load increases. The test is usually performed through several load-unload cycles. The data are subsequently analyzed to develop load-deformation curves from which a modulus can be obtained. The test is capable of loading a large volume of a rock mass from which a representative modulus can be obtained. The test, however, is extremely expensive. The test procedures are described in the Rock Testing Handbook (RTH-361).

g. Radial jacking tests. Radial jacking test is a modification of the pressure chamber test where pressure is applied through a series of jacks placed close to each other. While the jacking system varies, the most common system consists of a series of flat-jacks sandwiched between steel rings and the tunnel walls. The Rock Testing Handbook (RTH-367) describes the test procedures.

h. Borehole-jacking tests. Instead of applying a uniform pressure to the full cross-section of a borehole as in pressuremeter tests, the borehole-jack presses plates against the borehole walls using hydraulic pistons, wedges, or flatjacks. The technique allows the application of significantly higher pressures required to deform hard rock. The Goodman Jack is the best known device for this test. The test is inexpensive. However, the test influences only a small volume of rock and theoretical problems associated with stress distribution at the plate/rock interface can lead to problems in interpretation of the test results. For these reasons, the borehole-jacking tests are considered to be index tests rather than tests from which design moduli values can be estimated. The tests are described in the Rock Testing Handbook (RTH-368).

i. Tunnel relaxation tests. Tunnel relaxation tests involve the measurement of wall rock deformations caused by redistribution of in-situ stresses during tunnel excavation. Except for a few symmetrically shaped openings with known in-situ stresses, back calculations to obtain modulus values from observed deformations generally require numerical modeling using finite element or boundary element computer codes. The high cost of the test is associated with the expense of tunnel excavation.

4-22. Other Methods for Estimating Modulus

In addition to test methods in which modulus values are derived directly from stress-strain responses of rock, there are at least two additional methods frequently used to obtain modulus values. The two methods include seismic and empirical methods.

a. Seismic methods. Seismic methods, both downhole and surface, are used on occasion to determine the in-situ modulus of rock. The compressional wave velocity is mathematically combined with the rock's mass density to estimate a dynamic Young's modulus, and the shear wave velocity is similarly used to estimate the dynamic rigidity modulus. However, since rock particle displacement is so small and loading transitory during these seismic tests, the resulting modulus values are nearly always too high. Therefore, the seismic method is generally considered to be an index test. EM 1110-1-1802 and Goodman (1980) describe the test.

b. Empirical methods. A number of empirical methods have been developed that correlate various rock quality indices or classification systems to in-situ modulus. The more commonly used include correlations between RQD, RMR and Q.

(1) RQD correlations. Deere, Merritt, and Coon (1969) developed an empirical relationship for the in-situ modulus of deformation according to the following formula:

$$E_d = [(0.0231)(\text{RQD}) - 1.32]\ E_{t50} \qquad (4\text{-}5)$$

where

E_d = in-situ modulus of deformation

RQD = Rock Quality Designation (in percent)

E_{t50} = laboratory tangent modulus at 50 percent of the unconfined compressive strength

From Equation 4-5 it can be seen that the relationship is invalid for RQD values less than approximately 60 percent. In addition, the relationship was developed from data that indicated considerable variability between in-situ modulus, RQD, and the laboratory tangent modulus.

(2) RMR correlations. A more recent correlation between in-situ modulus of deformation and the RMR Classification system was developed by Serafim and Pereira (1983) that included an earlier correlation by Bieniawski (1978).

$$E_d = 10^{\frac{\text{RMR}-10}{40}}$$

where

E_d = in-situ modulus of deformation (in GPa)

RMR = Rock Mass Rating value

Equation 4-6 is based on correlations between modulus of deformation values obtained primarily from plate bearing tests conducted on rock masses of known RMR values ranging from approximately 25 to 85.

(3) *Q* correlations. Barton (1983) suggested the following relationships between in-situ modulus of deformation and *Q* values:

$$E_d\ (\text{mean}) = 25 \log Q \qquad (4\text{-}7a)$$

$$E_d\ (\text{min.}) = 10 \log Q \qquad (4\text{-}7b)$$

$$E_d\ (\text{max.}) = 40 \log Q \qquad (4\text{-}7c)$$

where

E_d (mean) = mean value of in-situ modulus of deformation (in GPa)

E_d (min.) = minimum or lower bound value of in-situ modulus of deformation (in GPa)

E_d (max.) = maximum or upper bound value of in-situ modulus of deformation (in GPa)

Q = rock mass quality value

4-23. Considerations in Selecting Design Modulus Values

Modulus values intended to be representative of in-situ rock mass conditions are subject to extreme variations. There are at least three reasons for these variations: variations in modulus definitions, variability in the methods used to estimate modulus, and rock mass variability.

a. Variations in modulus definitions. As noted in paragraph 4-20, the stress-strain responses of rock masses are not linear. Hence, modulus values used in design are dependent upon the portion of the stress-strain curve considered. Because the modulus of deformation incorporates all of the deformation behavior occurring under a given design stress range, it is the most commonly used modulus in analytical solutions for deformation.

b. Variability in methods. Modulus values obtained from tests are not unique in that the value obtained depends, for the most part, on the test selected. There are at least two reasons for this non-uniqueness. First, with the exception of laboratory compression tests, all of the methods discussed above are in-situ tests in which modulus values are calculated from suitable linear elastic solutions or represent correlations with modulus values derived from in-situ tests. Therefore, the validity of a given method depends to some extent on how well a given solution models a particular test. Finally, the volume of rock influenced by a particular test is a significant factor in how well that test reflects in-situ behavior. Recognizing the potential variation in modulus determinations, the plate-load test has become the most commonly used test for deriving the in-situ modulus of deformation for those projects requiring confidence in estimated values representative of in-situ conditions.

c. Rock mass variability. Deformational predictions of foundation materials underlying major project structures such as gravity and arch dams may require analytical solutions for multilayer media. In this respect, the selection of appropriate design deformation moduli will require consideration of not only natural variability within rock layers but also variability between layers.

4-24. Selection of Design Moduli

As in the selection of design shear strengths, the moduli values used for design purposes are selected rather than determined. The selection process requires sound engineering judgment by an experienced team of field and office geotechnical professionals. However, unlike shear strength selection, in which both upper and lower bounds of strength can generally be defined, only the upper bound of the deformation modulus can be readily predicted. This upper bound is derived from unconfined compression tests on intact rock. In addition, the natural variability of the foundation rock as well as the variability in derived modulus values observed from available methods used to predict modulus, complicates the selection of representative values of modulus. For these reasons, the selection process should not rely on a single method for estimating modulus, but rather the selection process should involve an intergrated approach in which a number of methods are incorporated. Index tests, such as the laboratory unconfined compression test and borehole test devices (Goodman jack, pressuremeter, and dilatometers), are relatively inexpensive to perform and provide insight as to the natural variability of the rock as well as establish the likely upper bounds of the in-situ modulus of deformation. Empirical correlations between the modulus of deformation and rock mass classification systems (i.e. Equations 4-5, 4-6, and 4-7) are helpful in establishing likely ranges of in-situ modulus values and provide approximate values for preliminary design. Index testing and empirical correlations provide initial estimates of modulus values and form the bases for identifying zones of deformable foundation rock that may adversely effect the performance of project structures. Sensitivity analyses, in which initial estimates of deformation moduli are used to predict deformation response, are essential to define zones critical to design. The design of structures founded on rock judged to be critical to performance must either reflect increasing conservatism in the selected modulus of deformation values or an increase in large scale in-situ testing (i.e. plate bearing tests, etc.) to more precisely estimate in-situ moduli. The high cost of in-situ tests generally limits the number of tests that can be performed. In this respect, it may not be economically feasible to conduct tests in rock representative of all critical zones; particularly for large projects founded on highly variable rock. In such cases site-specific correlations should be developed between the modulus of deformation values derived from both borehole index tests and large scale in-situ tests and rock mass classification systems (i.e. either the RMR system or the Q-system). If care is taken in selecting test locations, such correlations provide a basis for extrapolating modulus of deformation values that are representative of a wide range of rock mass conditions.

Section V
Use of Selected Design Parameters

4-25. General

For use of the selected design parameters, refer to the appropriate chapters as follows:

a. Chapter 5 - Deformation and Settlement (modulus of deformation).

b. Chapter 6 - Bearing Capacity (shear strength).

c. Chapter 7 - Sliding Stability (shear strength).

d. Chapter 8 - Cut Slope Stability in Rock (shear strength).

e. Chapter 9 - Anchorage Systems (shear strength).